Cardno
Shaping the Future

Ripley Valley Secondary Urban Centre East - Stages 2 and 3

Stormwater Management Report

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1 INTRODUCTION

This Stormwater Management Plan has been prepared by Cardno in relation to the land that is to accommodate the Secondary Urban Centre East Stage 2 (2A & 2B) and Stage 3 (3A, 3B & 3C), which is a part of the early release precinct in the Ripley Valley Urban Development Area.

This report describes the analyses and sizing work undertaken with respect to the following issues:

- stormwater quantity management; and
- stormwater quality management.

2 DESIGN CRITERIA

The *Ripley Structure Plan* (November 2007) presents a number of design criteria with respect to Water Cycle Management. Those criteria relating to stormwater management and flooding are as follows (Section 10.1.3.1):

- **Stormwater - Flood Management**

  - no increase in the flow discharged to Bundamba Creek, for the full range of design flood events (100-year average recurrence interval). Control measures will be required to attenuate the discharge from the developed areas. Measures may include:
    - detention basins (refer Figure 10.1.3); and
    - WSUD measures focussing on systems that maintain the existing times of concentration within sub catchments (i.e. retention of natural flow paths or use of swales where practicable rather than a piped drainage system to detach impervious areas).
  - existing watercourses to be maintained for conveyance of stormwater runoff where practicable, especially in upper reaches of the catchments where no existing 100 year flood inundation has been defined;
  - protection of stream ecology and geomorphology from impacts of stormwater runoff;
  - adoption of Healthy Waterways WSUD Developing Design Objectives for Water Sensitive Developments in South East Queensland including no increase in the pre-development peak flow in watercourses from the 1 year event;
  - capture and manage the following design runoff capture depth (mm/day) from all impervious surfaces:
    - 0 to 5% impervious: No capture required.
    - 5 to 20% impervious: Capture first 5mm/d of runoff.
    - 20 to 60% impervious: Capture first 10mm/d of runoff.
    - >60% impervious: Capture first 15mm/d of runoff.

- **Stormwater - Quality Management**

  Water quality objectives will be in accordance with South East Queensland Healthy Waterways publication *Water Sensitive Urban Design, Developing Design Objectives for Water Sensitive Urban Development in South East Queensland* (October 2006) best practice treatment design objectives as follows:

  - 80% reduction in sediment load;
  - 60% reduction in total phosphorus load;
  - 45% reduction in total nitrogen load; and
  - 90% reduction in gross pollutant load.

No use is to be made of existing watercourses or waterway vegetation for the treatment of runoff. It is necessary to treat all runoff from development areas discharging directly to second order streams identified in the planning process.

It is envisaged that the form of first order streams within identified development areas will be significantly modified by development and, therefore, it is not proposed to require runoff directed to first order streams to be treated to meet the water quality objectives. The only exception to this would occur if the stream were to be retained as a feature of the area under development, in which case the water objectives would be applicable.

This report describes the investigations and preliminary design undertaken to confirm that the Secondary Urban Centre East can satisfy the above criteria.

3 STORMWATER QUANTITY

3.1 General

In order to comply with the design criteria for Ripley Valley (refer Section 2), it will be necessary to construct a detention basin to temporarily store runoff and reduce the peak flow discharge for the Secondary Urban Centre East.

The proposed location for the detention basin is shown in Figure 1.

The requirements for the detention basin were determined by dynamic modelling.

3.2 Hydrology

3.2.1 Rainfall, Losses and Temporal Patterns

Appropriate rainfall data for the area was determined in accordance with the procedures and parameters provided in Volumes 1 and 2 of *Australian Rainfall and Runoff, A Guide to Flood Estimation* (Institution of Engineers Australia, 1987). The design rainfall intensities for Ripley are listed in Table 1.

Table 1 Design Rainfall Intensities

Duration	Rainfall Intensity (mm/h)						
	1 Year	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
15 Min	71	92	117	132	153	181	202
20 Min	62	80	102	116	134	159	178
25 Min	55	72	92	104	120	143	160
30 Min	50	65	84	95	110	130	146
45 Min	40.4	52	67	77	89	106	119
1 Hour	34.4	44.5	58	66	76	91	102
90 Min	25.5	33	42.8	48.9	57	68	76
2 Hour	20.5	26.6	34.6	39.5	46.1	55	62
3 Hour	15.1	19.6	25.5	29.2	34.1	40.7	46
4.5 Hour	11	14.4	18.8	21.6	25.2	30.2	34.1
6 Hour	8.86	11.5	15.1	17.4	20.3	24.4	27.6
9 Hour	6.5	8.48	11.2	12.8	15.1	18.1	20.5
12 Hour	5.22	6.82	9	10.4	12.2	14.6	16.6
24 Hour	3.29	4.3	5.67	6.54	7.68	9.23	10.5
36 Hour	2.49	3.25	4.29	4.94	5.81	6.98	7.92
72 Hour	1.48	1.93	2.55	2.94	3.45	4.15	4.71

The initial and continuing losses adopted for the catchment were as follows:

- *Pervious Areas:* No initial loss and 2.5mm/h continuing loss
- *Impervious areas:* No initial or continuing loss

For the analysis, events with storm durations between 15 minutes and 6 hours were considered. Temporal patterns for Zone 3 were adopted as appropriate for each duration in accordance with Volume 2 of *Australian Rainfall and Runoff* (Institution of Engineers Australia, 1987).

3.2.2 Rational Method Calibration

In order to confirm that the peak flows predicted by the hydrologic models were reliable, the model was calibrated to the peak flows calculated using the Rational Method in accordance with the *Queensland Urban Drainage Manual* (2007) (QUDM).

The time of concentration of each catchment was calculated as described below:

- **Existing Case**

  The initial time of concentration was calculated using Friend's equation for the first 200 metres of overland flow (Table 4.06.3 of QUDM). Beyond this point, flow is concentrated and the average stream velocities presented in Table 4.06.5 of QUDM were used to calculate travel times.

- **Developed Case**

  For the developed case, standard inlet times in accordance with Table 5.05.1 of QUDM were adopted. Runoff was then assumed to be piped at a velocity of 3 m/s.

Runoff coefficients were derived from Table 4.05.3 of QUDM based on a one hour 10 year rainfall intensity of between 65 and 69 mm/h. For existing areas, a fraction impervious of zero and a runoff coefficient for the 10 year event of 0.66 was adopted. For the developed case, a fraction impervious of 0.8 and a runoff coefficient of 0.85 was adopted for the Secondary Urban Centre East. A frequency factor of 1.2 (QUDM Table 4.05.2) was applied to the 10 year runoff coefficient to derive the 100 year runoff coefficient. As recommended in QUDM, in cases where the calculated 100 year runoff coefficient was greater than 1, a value of 1 was adopted.

The parameters adopted and the peak flows derived for each subcatchment are listed in Table 2.

Table 2 Peak Flow Calculation – 100 year Event

Node or Point of Interest	Rational Method				SWMM/RAFTS/WBNM		
	Time of Concentration (min)	Rainfall Intensity (mm/h)	Area (ha)	Flow Rate (m^3/s)	Peak Flow (m^3/s)	Critical Duration (min)	Diff. (%)
Existing Case (Site Undeveloped)							
N-SC2-01a	Friend's 19 min (n=0.06, slope 4%), then av. Velocity of 0.9 m/s for 843 m Total 34min	137	37.82	11.40	11.45	60	0.5
N-SC2-01b	Friend's 18 min (n=0.06, slope 5.5%), then av. Velocity of 0.7 m/s for 387 m Total 24 min	160	17.12	6.03	6.05	60	0.3
N-SC2-02	Friend's 24.8 min (n=0.06, slope 7.9%), then av. Velocity of 0.9m/s for 255 m and 0.7 m/s for 350 m and 0.3 m/s for 155 m, Total 46.4 min	117	25.86	6.66	6.69	60	0.5
N-SC2-03	Friend's 22.2 min (n=0.06, slope 13.7%), then av. Velocity of 1.5 m/s for 175 m and 0.3 m/s for 245 m Total 37.6 min	131	13.69	3.95	3.98	60	0.7
Developed Case							
N-SC2-01a	10 min (Standard inlet time, 9% slope) plus 943 m piped at 3 m/s, Total = 15 min	202	37.82	21.65	21.71	25	0.3
N-SC2-01b	10 min (Standard inlet time , 8% slope) plus 487 m piped at 3 m/s, Total = 13 min	189	17.12	10.43	10.22	25	-2.0
N-SC2-02	10 min (Standard inlet time, 8% slope) plus 835 m piped at 3 m/s and 155 m overland at 3.0 m/s, Total = 23.2 min	165	22.31(1) 3.55(0.79)	11.5	11.61	25	1.0
N-SC2-03	8 min (Standard inlet time, 14% slope) plus 375 m piped at 3 m/s and 245 m overland at 0.3 m/s, Total = 23.6 min	164	11.94(1) 4.97(0.79)	7.24	7.59	25	4.8

Notes: For location of nodes and contributing catchments, refer Figure 2 and figure 3.
Areas with more than one applicable runoff coefficient presented as area (100 year runoff coefficient).
Peak flow values followed by (Partial) have peak flow for only part of are contributing

3.2.3 SWMM Modelling of Runoff

The Secondary Urban Centre East detention basin (Refer to Figure 1) was modelled using the dynamic model XP-SWMM. This model consists of runoff and hydraulic layers. The runoff layer was used to calculate runoff hydrographs for the subcatchments defined for each model. The Laurenson method, as implemented in the RAFTS program, was used to calculate runoff hydrographs for a range of durations and events.

Each subcatchment was divided into pervious and impervious areas based on fraction impervious values which in turn were derived from the runoff coefficient adopted for each landuse within each subcatchments and the correlation between fraction impervious and runoff coefficient presented in the *Queensland Urban Drainage Manual*. As the runoff layer allows the specification of up to 5 subcatchments per node, areas of open space were modelled separately from the previous component of developed areas.

The Laurenson (RAFTS) method requires that the Mannings n and slope of each subcatchment be defined. Suitable values for these parameters were adopted for each subcatchment based on the recommendations of the SWMM User Manual (XP-Software) and existing topography. The value of slope was adjusted as a calibration parameter.

The parameters ultimately adopted for the investigation are listed in Table 3.

Table 3 SWMM Model Runoff Parameters

Node	Area (ha)	Pervious Subarea			Impervious Subarea		
		Area (ha)	n (-)	Slope (m/m)	Area (ha)	n (-)	Slope (m/m)
Existing Case *(Site Undeveloped)*							
N-SC2-01a	37.82	37.82	0.05	0.06	-	-	-
N-SC2-01b	17.12	17.12	0.05	0.062	-	-	-
N-SC2-02	25.86	25.86	0.05	0.035	-	-	-
N-SC2-03	13.69	13.69	0.05	0.035	-	-	-
Developed Case *(Site Developed and Catchment Areas Increased)*							
N-SC2-01a	37.82	7.56	0.05	0.005	30.26	0.025	0.005
N-SC2-01b	17.12	3.42	0.05	0.02	17.846	0.025	0.002
		3.552	0.05	0.005	-	-	-
N-SC2-03	16.88	2.388	0.05	0.005	9.522	0.025	0.005
		4.97	0.05	0.009	-	-	-

Note: *Refer to Figure 1 for location of nodes*
Pervious nodes assumed to have nil impervious area, impervious nodes 100 % impervious area.

The Laurenson method uses the parameter B to reflect the storage available within subcatchments. The parameters can be modified by a multiplier Bx as part of the calibration process. The Bx value was adjusted until a suitable agreement was obtained between the peak flow predicted using the SWMM model and that calculated using the Rational Method. A Bx value of 1.25 was found to provide the best agreement to the Rational Method flows and was adopted for the analysis. The agreement was then fine turned by adjusting the slope and Mannings n (PERN) roughness parameters for each subcatchment.

The peak flow rates obtained from the SWMM model and the Rational Method are presented in Table 3. Given the close agreement in values, the runoff calculated using the SWMM (RAFTS) runoff algorithms can be applied with confidence to the modelling of the detention basin.

It can be noted that the calibration was completed for the 100 year event. For lesser events, the SWMM model will predict peak flow rates higher than those calculated using the Rational Method. This is due to the fact that the runoff coefficient used in the Rational Method decreases as more frequent events are considered. Unless the losses adopted for the SWMM model are increased, the SWMM model will estimate peak flows greater than those calculated using the Rational Method.

However, as the subject of the modelling was a detention basin, it was considered preferable to maintain the losses adopted for the 100 year event in order to ensure that the volume of runoff occurring for lesser events was not underestimated.

3.3 Hydraulics

3.3.1 General

The Secondary Urban Centre East detention basin was modelled using the dynamic model XP-SWMM.

3.3.1.1 SWMM Modelling

3.3.1.2 Model Setup

The hydraulic layer of SWMM was used to model the existing and developed cases, and to determine the required dimensions of the detention basin required for the Secondary Urban Centre East (Refer to Figure 1 for location of basin).

The layout of the SWMM models is shown on Figure 1.

Cross sections for the model were extracted from available aerial survey data. Based on a site inspection and aerial photography, a Mannings n value of 0.05 was generally adopted. In areas where planting is proposed within the developed channel and basin areas a Mannings n value of 0.08 was adopted.

Version 2009 service pack 3 of the software was used for the analysis. To ensure model stability, a timestep of 10 seconds was adopted for the analysis.

As the critical storm duration (i.e. the duration producing the peak flow) will occur well before the peak flow in the Bundamba Creek, a tailwater condition based on normal flow depth was adopted at the outlet of each basin.

3.3.1.3 Detention Basin Sizing

The model was first run to establish the existing case results against the impact of development could be assessed. For the analysis, storm durations between 15 minutes and 6 hours were considered for events with recurrence intervals between 1-100 years.

The results obtained for the existing case are presented in Appendix A.

It is proposed that a combination of a drainage channel, detention basin and neighbouring sports field be used to achieve sufficient storage volume while minimising the depth of storage in the basin. Low flow pipes and a weir will be located at the downstream end of the detention basin to limit discharge.

The area adopted for each basin were varied until a volume sufficient to reduce the peak flow from the developed case equal to or less than the peak flow predicted for the existing case was obtained for all events from the 1 to 100 year events inclusive.

A channel is proposed upstream of the main drainage channel between Barrams Road and the new loop road. The channel will convey runoff from subcatchments N-SC2-01a and N-Sc2-01b to culverts under the new road which drains flow to the main drainage channel and then into the detention basin. The channel details are shown in Table 4.

A drainage channel is proposed downstream of the new road to convey flow from the culverts into the detention basin. A drop structure is positioned 73.5 metres downstream of the culverts to reduce the slope of the channel and allow a portion of the downstream reach of the channel to be used for flood storage. The channel details are shown in Table 4.

A detention basin is proposed downstream of the drainage channel. The basin has been sized to attenuate runoff from Secondary Urban Centre East 2 (SUCE 2) catchments. The sports field positioned alongside the detention basin will also provide additional attenuation for events greater than the 10 Year ARI event. Details of the proposed basin and sports field as shown in Table 4.

Table 4 Secondary Urban Centre East Channel and Basin Details

Basin Parameter	Drainage Channel	Culverts Beneath Proposed Road	Main Drainage Channel		Sports Field	Detention Basin
			Section A	Section B		
U/S Invert Level (mAHD)	53.5	47.49	46.0	42.5	43.9	42.
D/S Invert Level (mAHD)	47.49	46.0	43.5	42.0	43.1	41.7
Slope (m/m)	Varies (1/40-1/60)	1/15	1/30	1/500	1/200	1/500
Roughness (n)	0.08	0.014	0.08	0.08	0.045	0.08
Base Dimensions (m)	Varies	*na	20.6	20.6	140	147
Depth Including Freeboard (m)						1.8
Top Area (1 in 4 bank Slopes) (ha)	*na	*na	4	4	4	4
Outlet Configuration		7/1.2*0.9 RCBC				Weir – 30.m long crest at 42.7 Low Flow – 7/825mm RCP

The results obtained for the developed case with the proposed attenuation are listed in Appendix A. The peak flow rates obtained at the point of discharge to Bundmba Creek from the detention basin are summarised in Table 5.

Table 5 Summary of Detention Basin Discharge

Event	Peak Discharges (m^3/s)	
	Existing Case	Developed Case
1	6.0	5.5
2	8.4	6.1
5	11.6	7.8
10	14.6	10.2
20	18.7	13.6
50	23.0	18.5
100	26.5	22.5

With reference to the results presented in Table 5, it can be concluded that the proposed detention basin in combination with the drainage channel and sports field will effectively reduce the peak flow discharge from the developed catchment to match that produced by the existing (undeveloped) catchment.

4 CONSTRUCTION PHASE

4.1 General

During the construction phase, the potential exists for increases in the amount of pollutants, particularly sediment, exported from the site. Throughout the construction phase, an Erosion and Sediment Control Plan will be required as part of the overall Environmental Management Plan.

The objective of the plan will be to control soil erosion on the site and prevent sediment discharge to downstream local water courses during storm events for the construction phase of the project.

4.2 Water Quality Objectives

The National Water Quality Management Strategy (which is annexed by the Environmental Protection Act (1994) and subordinate Environmental Protection (Water) Policy (1997)) indicates that increases in suspended particulate should be limited such that the mean turbidity of receiving waters, either fresh or marine, does not increase by more than 10%. In general terms, the concentration of suspended sediment discharged from the site should not exceed 50mg/L.

In conjunction with the above, the local Council (Ipswich City Council) recommends that construction phase concentrations should be in accordance with *Best Practice Erosion and Sediment Control* published by the International Erosion and Sediment Control Association Australasia. It is considered that this represents best practice and has been adopted for the purposes of the preparation of the Stormwater Management Plan.

Based on the above information, the release criteria for pumped discharges from the construction site shall be as shown in Table 6 below.

Table 6 Construction Phase Water Quality Objectives

Parameter	Release Criteria	Criteria Type
pH	6.5 – 9.0	Range
Suspended Solids	<50 mg/L	Maximum
Dissolved Oxygen	>6 mg/L	Minimum
Hydrocarbons	No visible sheen on receiving water	Descriptive
Litter	No visible litter washed from site	Descriptive

4.3 Erosion and Sediment Control Measures

Although a detailed Erosion and Sediment Control Plan will be prepared as part of detailed design, consideration was given to the likely method of water quality management during the construction phase. At this point, it is envisaged that runoff from disturbed areas will be directed to a sediment basin. It is envisaged that a basin will be located at the upstream end of the road crossing and adjacent to the main drainage channel (location of the basin shown in Figure 2). The basin will be removed following the completion of construction activities (when operational phase devices are commissioned).

For stages 2 and 3 it is proposed to provide a sedimentation basin on the upstream side of the road crossing to capture runoff from the catchment. The location of the basin is shown in Figure 2.

The required size of the sediment basin was calculated. It was conservatively judged that the basin will be at its design limit when fifty percent of the catchment is sealed surface. Due to the soil within the catchments being more than 10 percent dispersible, Type D basins have been designed for both catchments. The basin will be removed once construction of the stages has finished.

The sizing derived for the sediment basin is summarised in Table 7.

Table 7 Construction Phase (Temporary) Sediment Basin Sizing

Parameter	Sediment Basin (Stages 2A, 2B,3A, 3B and 3C)
Catchment Area (ha)	15.6
Volumetric Runoff Coefficient (Cv)	0.75
Intensity (1yr24hr)	3.29
Settling Volume (m^3)	3850
Setting Depth (m)	1.5
Sediment Storage Volume (m^3)	1925

The basin has been sized to allow for the sediment to sufficiently settle out of the captured stormwater and will not require the use of chemical flocculation agents to assist in the process.

5 OPERATIONAL PHASE

5.1 Water Quality Objectives

Relevant water quality objectives were defined based on the best practice guidelines nominated in the Department of Infrastructure and Planning *South East Queensland Regional Plan 2009-2031, Implementation Guideline No.7* (November 2009).

The guidelines consider three criteria, as described below.

- **Stormwater Quality Management**

  The intent of the criterion is to reduce the impact of urban development on pollutant loads discharged to receiving waters by achieving the following minimum reductions in total pollutant load compared with that of untreated stormwater runoff:

  - 80 percent reduction in total suspended solids;
  - 60 percent reduction in total phosphorus;
  - 45 percent reduction in total nitrogen; and
  - 90 percent reduction in gross pollutants.

- **Waterway Stability Management**

  The intent of this criterion is to reduce the impact of urban development on channel bed and bank erosion by limiting the post-development one year average recurrence interval event discharge within the receiving waters to the pre-development peak.

  In this case, the stage under consideration is located immediately adjacent to Bundamba Creek. Ultimately, detention basins will be provided to service the SUCE development as a whole and achieve the criterion for the development as a whole. For the first stage of the development, it is considered that the increase in peak flow in Bundamba Creek as a result of development of Stage 1 will be negligible and that therefore it is not proposed to adopt the criterion for waterway stability for Stage 1.

- **Frequent Flow Management**

  The intent of this criterion is to reduce the impact of development on the frequency at which runoff occurs. To satisfy this criterion, it is necessary to capture and manage the first 10 mm of runoff from impervious surfaces when the fraction impervious of a catchment is less than 40 percent, and to capture and manage the first 15 mm of runoff from impervious surfaces when the fraction impervious of a catchment is greater than 40 percent.

5.2 Available Treatment Measures And Their Suitability

A wide range of stormwater management practices are available to achieve the principles of Water Sensitive Urban Design. All have been shown to be successful when correctly designed, however, selection of the most appropriate practices for a particular development is highly dependent on site conditions.

The table below lists the most common stormwater management practices currently used, along with opportunities and constraints for their use. This table forms the starting point for stormwater management for the development. Considering the site's constraints, not all of the measures outlined in the table are suitable. Each treatment option is therefore discussed in more detail and comment made on the suitability of each in the context of the development.

Table 8 Stormwater Management Practices

Practice	Opportunities			Constraints
	Pollutants removed	Scale	Other	
Litter baskets/racks	Litter and debris – M	Local	Pre-treatment for other practices	Require frequent maintenance
Sediment traps	Coarse sediment – H Suspended solids – L	Regional		Aesthetic and safety issues
Gross pollutant traps (GPTs)	Litter and debris – M Coarse sediment – H Suspended solids – L	Regional	Pre-treatment for other practices	Require regular maintenance
Filter strips/ buffer strips	Litter and debris – M Coarse sediment – H Suspended solids – H Nutrients – M	Lot/Local	Peak flow management	Require flat terrain
Grass swales	Litter and debris – M Coarse sediment – H Suspended solids – H Nutrients – M	Local	Peak flow management	Require flat terrain
Vegetated swales	Litter and debris – M Coarse sediment – H Suspended solids – H Nutrients – H	Local	Peak flow management	Require flat terrain
Extended detention basins	Coarse sediment – M Suspended solids – M Nutrients – L	Regional	Peak flow management	Requires pre-treatment Large land area required
Infiltration trenches	Suspended solids – H Nutrients – M	Local	Peak flow management	Requires pre-treatment
Bio-retention systems	Coarse sediment – H Suspended solids – H Nutrients – H	Local	Peak flow management	Requires pre-treatment
Roof Gardens	Coarse sediment – H Suspended solids – H Nutrients – H	Local	Peak flow management	Probably not practicable for individual houses
Porous pavements	Suspended solids – H Total phosphorus – M Total nitrogen – L	Local	Peak flow management	Not appropriate for steep sites
Constructed wetlands	Coarse sediment – H	Regional	Peak flow	Requires pre-treatment

Practice	Opportunities			Constraints
	Pollutants removed	Scale	Other	
	Suspended solids – H Total phosphorus – M Total nitrogen – L		management	Not appropriate for steep sites Requires large area of land
Rainwater tanks	N/A	Lot	Peak flow management	Mandatory
Community education	Litter and debris – L Nutrients – L	Regional	Other benefits include weed control, reduced water use	Community participation

Note: Data taken from NSW EPA Managing Urban Stormwater – Treatment Techniques, 1997
L – 10 – 50% removal
M – 50 – 75% removal
H – 75 – 100% removal
Scales
Lot less than 1 ha
Local 1 – 10 ha
Regional greater than 10 ha

Each of the techniques described in the previous table is further assessed below, with regard to the site's specific opportunities and constraints.

Litter Baskets/racks

The primary purpose for litter baskets is to remove medium sized litter and debris from the site. Apart from high density areas, significant litter generation is not expected. It is therefore suggested that consideration be given to the use of litter baskets in gully pits as necessary within the development.

Sediment Traps

Sediment traps will be required during the construction phase to limit the transport of sediment off site. Following construction, sediment levels will return to a low level, and traps are therefore not considered necessary as a long-term water quality measure.

Gross Pollutant Traps

Gross pollutant traps (GPTs) are predominantly used for the removal of litter and hydrocarbons – although they have been shown to effectively remove coarse sediment and suspended solids (Brisbane City Council, 1999). It is expected that GPT's would only be necessary where the potential exists for significant oil and grease deposition (eg car parks). In this case, it is proposed to use surface level bioretention systems to treat car park runoff, obviating the need for GPTs.

Filter Strips/Buffer Strips

Buffer strips are areas of land left in their natural state that act to reduce peak runoff flows and improve the quality of stormwater runoff. This treatment improves the aesthetic and biodiversity value of the development and provides significant quality and quantity improvements.

Grass Swales

Grass swales are considered a highly effective and aesthetic water quality (and to a lesser extent, water quantity) control measure, when placed in flat areas. Given the slopes that occur across the catchment, the opportunities for the use of swales within the development footprint may be limited.

Vegetated Swales

Vegetated swales require similar conditions as those outlined for grass swales above, and wherever slopes are sufficiently low they would provide effective stormwater treatment for the site.

Extended Detention Basin

Extended detention basins are designed to generally store runoff for 1-2 days. Their main purpose is the reduction of the peak discharge from the site during a storm event, and the retention of particulate matter. Detention basins will ultimately be included across the development. However, the basins will not store runoff for such a period and so will not function as extended detention basins.

Infiltration Trenches

Infiltration trenches allow pre-treated stormwater runoff to infiltrate into surrounding soils and groundwater, and as such are not a treatment device in themselves. Where space is limited, bio-retention systems are preferred.

Bio-retention Systems

Bio-retention systems effectively combine a grass or vegetated swale with an infiltration trench. They are considered to be extremely effective in removing sediment and nutrients. Bio-retention basins will be employed across the development, located in the base of the aforementioned detention basins in order to treat stormwater to an appropriate standard prior to release.

Roof Gardens

Roof gardens offer the potential to capture and treat incident rainfall at source, offering benefits in terms of peak flow reduction and amenity if used as an integral part of a building (for instance as part of a roof top recreation/ lunch area). Their use would be encouraged for suitable developments. However, the majority of the stage contains residential properties, for which the use of roof gardens is not considered to be viable.

Porous Pavements

Porous pavements are not appropriate for high traffic areas due to increased maintenance requirements. They are also less effective in steep areas. However, for relatively flat, low traffic roads they could be suitable for this development, although their perceived expense often leads to the preference of other measures in reaching quantity and quality objectives.

Constructed Ponds and Wetlands

Wetlands require reasonably large flat areas of land. The potential exists to incorporate relatively large wetlands in the lower parts of the Secondary Urban Centre East or to distribute wetlands within urban areas at suitable locations.

Rainwater Tanks

The ability exists to install rainwater tanks throughout the development to reduce overall water consumption. This is an important component of the overall water cycle management strategy for the valley.

5.3 Adopted Treatment Measures

Based on the review of available treatment options, the following treatment measures have been adopted for Stages 2 and 3 (2A, 2B, 3A, 3B and 3C) of the development.

- **Rainwater tanks**

  Rainwater tanks will be constructed on each residential lot in accordance with the requirements of the Queensland Development Code to collect roofwater runoff from roof areas for non-potable reuse (toilet flushing, laundry use and irrigation).

- **Bioretention basin**

  Runoff from low flow events will be directed to a bioretention basin for treatment. Runoff from larger events would bypass the basin. The basin has been sized to meet the frequent flow requirements. A Sediment forebay will be provided at the point of entry to the basin to allow for the removal of coarse sediment.

5.4 MUSIC Model

A MUSIC (Model for Urban Stormwater Improvement Conceptualisation, CRC for Catchment Hydrology, Version 3.0.1) model of the development was established to assess the quality of the stormwater discharge from the site.

In accordance with current best practice, the site was divided into various components for the purposes of modelling:

- Roof;
- Road;
- Commercial; and
- Landscaped Areas

Appropriate rainfall-runoff and pollutant export parameters for the site were derived in accordance with the recommendations of the Gold Coast City Council *MUSIC Modelling Guidelines* (2006). The parameters adopted for each source node are listed below in Table 9 and Table 10.

Six minute rainfall data from Amberley for the 10 year period from 1990 to 1999 was used for the analysis. This information was obtained from the metrological database provided with MUSIC.

Table 9 Rainfall Runoff Parameters

Parameter	Urban Node (Roof Area)	Urban Node (Road Area)	Urban Node (Commercial Area)	Urban Node (Landscaped)
Rainfall threshold (mm)	1.0	1.0	1.0	1.0
Soil storage capacity (mm)	400.0	400.0	120.0	400.0
Initials storage (% capacity)	10.0	10.0	25.0	10.0
Field capacity (mm)	200.0	200.0	80.0	200.0
Infiltration capacity coefficient a	50.0	50.0	200.0	50.0
Infiltration capacity exponent b	1.0	1.0	1.0	1.0
Initial depth (mm)	50.0	50.0	50.0	50.0
Daily recharge rate (%)	25.0	25.0	25.0	25.0
Daily baseflow rate (%)	5.0	5.0	5.0	5.0
Daily deep seepage rate (%)	0.0	0.0	0	0.0

Table 10 Pollutant Export Parameters

Catchment ID	Land Use	Flow Type	TSS Log_{10} Values		TP Log_{10} Values		TN Log_{10} Values	
			Mean	St.Dev	Mean	St.Dev	Mean	St.Dev
Site	Roof	Baseflow	1.00	0.34	-0.97	0.31	0.20	0.20
		Stormflow	1.30	0.39	-0.89	0.31	0.23	0.23
	Road	Baseflow	1.00	0.34	-0.97	0.31	0.20	0.20
		Stormflow	2.43	0.39	-0.30	0.31	0.26	0.23
	Urban	Baseflow	1.00	0.34	-0.97	0.31	0.20	0.20
		Stormflow	2.18	0.39	-0.47	0.31	0.26	0.23
	Commercial	Baseflow	0.78	0.39	-0.60	0.50	0.32	0.30
		Stormflow	2.16	0.38	-0.39	0.34	0.37	0.34

The catchment was divided up into a number of subareas, as shown in Figure 3. The size and description of each land use within each subarea is listed in Table 11. Based on the lots size, a roof area equivalent to 50% of the total lot area was adopted. Of this roof area, it was conservatively assumed that only half would drain to a rainwater tank. The remaining roof area was assumed to drain to the treatment device being modelled. Roof areas were assumed to be completely impervious.

The remainder of the lot was included in the urban node with an assumed to be 50% impervious (to account for pathways etc).

Table 11 Music Model Areas

Catchment	Subarea	Description	Area (ha)	Fraction Impervious (%)
Stage 2A & 2B	Roof Area	Roof draining to tank	0.77	100
	Roof Area	Roof draining to treatment device	0.77	100
	Urban Area	Remainder of lot area	1.54	50
	Road Area	Road and road reserve areas	1.59	70
Stage 3A & 3B	Roof Area	Roof draining to tank	1.12	100
	Roof Area	Roof draining to treatment device	1.12	100
	Urban Area	Remainder of lot area	2.24	50
	Road Area	Road and road reserve areas	2.84	70
	Open Space	Area of district park and stormwater management	1.38	0
Stage 3C	Roof Area	Roof draining to tank	0.37	100
	Roof Area	Roof draining to treatment device	0.37	100
	Urban Area	Remainder of lot area	0.74	50
	Road Area	Road and road reserve areas	0.79	70

5.5 Treatment Device Design

5.5.1 Rainwater Tanks

As stated earlier it is proposed that rainwater tanks be installed within each residential allotment, capturing runoff from half of the roof area.

A daily water supply demand of 390 Litres per household has been adopted for MUSIC modelling. The daily water supply demand was determined assuming the following variables with each household within the proposed development.

- Consumption rate = 240 (L/person/day)
- Average household occupancy = 2.5 persons
- Breakdown of Residential Total Water Use
  - External Use 30% of total use
  - Toilets 20% of total use
  - Laundry 15% of total use

The daily water supply demand was therefore adopted to be 390 litres/household/day (65%*2.5*240 L/p/d) = 390 L/household/day).

The parameters adopted to model the rainwater tanks according to the *MUSIC Modelling Guidelines for South-East Queensland* are listed below in Table 12.

Table 12 Rainwater Tank Parameters

Parameters	Stage 2A & 2B	Stage 3A & 3B	Stage 3C
Volume below overflow pipe (kL)	320	455	155
Depth above overflow (m)	0.2	0.2	0.2
Surface area (m²)	160	228	78
Overflow pipe diameter (mm)	720	860	500
Daily Demand (kL/day)	24.96	35.49	12.09

5.5.2 Bioretention Basin

To satisfy the water quality objectives nominated in Section 3.1, it is proposed to construct a bioretention basin to service the contributing catchments. For Stages 2 and 3, a basin with a surface area of 1,100m² has been adopted. This basin will also provide treatment for the future stages of the development and therefore will be increased in size to adequately meet objectives as futures stages progress. The indicative location of the each of the basin is shown in Figure 3.

The properties of the bioretention system are presented in Table 13.

Table 13 Properties of Bioretention Basin

Parameters	Valve
Storage Properties	
Extended detention depth (m)	0.3
Surface Area (m²)	1100
Infiltration Properties	
Seepage Loss (mm/h)	0
Filter Area (m²)	960
Filter Depth (m)	0.6
Filter Median Particle Diameter (mm)	0.45
Saturated Hydraulic Conductivity (mm/h)	180

Runoff entering the basin due to overland flow will first be directed to a sediment forebay to allow the removal of course sediment.

The sizing of the forebay is summarised in Table 14.

Table 14 Sediment Forebay Details

Parameters	Basin 1
Contributing Catchment Area (ha)	15.56
Capture Efficiency (%)	80
Sediment Loading Rate (m³/ha/yr)	1.6
Desired Clean out Frequency (yr)	2
Volume of Forebay Sediment Storage Required (m³)	40
Depth of Sediment Forebay (m²)	0.3
Surface Area of Sediment Forebay (m²)	135

5.5.3 Frequent- Flow Management

As part of the stormwater management plan for the development, it is proposed to capture and manage the frequent flows generated from the impervious surfaces on the development site, in accordance with the *Concept Design Guidelines for Water Sensitive Urban Design Version 1 (March 2009).*

According to these guidelines, for a development that has an impervious fraction of greater than 40%, the first 15 mm of runoff from the impervious surfaces should be captured. This volume of storage should be available again within 24 hours.

In the case of the site, the storage will be achieved by reuse from the rainwater tanks and filtration through the bioretention system. Conservatively, the reuse available through the rainwater tanks proposed on each lot was not included in the analysis.

Consideration was therefore given to the storage available in the extended detention depth above the filter media in the bioretention system. It can be noted that the bioretention system will drain relatively rapidly as the saturated hydraulic conductivity of the filter media is 180 mm/h. This is faster than the drawdown rate required to drain the frequent flow storage in 24 hours. To assess the amount of runoff that passes through the bioretention system and whether the frequent flow criterion is satisfied, the methodology presented in the Bligh Tanner and DesignFlow publication *Stormwater Infrastructure Options to Achieve Multiple Water Cycle Outcomes* (report for Queensland Water Commission, 2009, p34):

- Use MUSIC to calculate the percentage of annual runoff that the first 15mm of runoff from impervious areas accounts for (for the 10 year rainfall record at Amberley);
- Use MUSIC to calculate the proportion of annual runoff that is treated by the bioretention system; and
- If the percentage of annual runoff that passes through the bioretention system is greater than the percentage of annual runoff that the first 15mm of runoff accounts for, then the frequent flow objective has been met.

Based on the MUSIC model described above, the following values were obtained:

- Average annual runoff form site — $760{,}040m^3$
- Average annual runoff produced by first 15mm of runoff from impervious areas — $399{,}082m^3$
- Proportion of annual runoff produced by the first 15mm of runoff from impervious areas — 60.2%
- Proportion of runoff treated by bioretention system — 68.3%

As the percentage of annual runoff that passes through the bioretention system is greater than the percentage of annual runoff that the first 15mm of runoff from impervious areas accounts for, it can be concluded that the frequent flow management criterion is satisfied.

5.5.4 Water Quality Modelling Results

The results of the MUSIC analysis are presented in Table 15.

Table 15 shows the quantities, expressed in kilograms each year, of pollutants exported from the site is the mitigated and proposed situations, and the percentage reduction afforded by the proposed treatment measures.

Table 15 Reduction in Total Annual Loads

Pollutant	Prior to Treatment (kg/y)	Post Treatment (kg/y)	Percentage Reduction (%)	Target Reduction (%)
Total Suspended Solids	15400	2810	81.8	80
Total Phosphorus	30.0	9.31	68.9	60
Total Nitrogen	158	83.7	47.1	45
Gross Pollutants	2180	0.0	100	90

With reference to Table 15 it can be concluded that the adopted water quality objectives will be achieved. A schematic of the bioretention basin is shown in Appendix B.

5.5.5 Waterway Stability Criteria

With reference to Section 4 of the report, the proposed detention basin will reduce the peak discharge from the site for the 1 year event to match that produced by the existing catchment, thereby satisfying the *Waterway Stability Criteria.*

5.6 Life Cycle Costs

A life cycle costing was completed with respect to the proposed works for consistency with the Stage 1 reporting.

The life cycle costs for the proposed rainwater tanks will be borne by the owner of the respective property, rather than by council. Therefore, the life cycle costs for the rainwater tanks are not shown below.

Life cycle costs for the proposed treatment train (i.e Bioretention Basins) in the eastern and western catchments were determined using the recommended parameters in MUSIC. These include:

- Real discount rate 5.5%
- Annual inflation rate 2.0%
- Base year for costing 2011
- Life Cycle 50 years
- Renewal/adoption period 20 years
- Annual maintenance cost MUSIC "expected value"
- Annualised renewal cost MUSIC "expected value"
- Decommissioning cost MUSIC "expected value"

The design and construction costs for the bioretention basin will be borne by the developer, not by Ipswich City Council (the ultimate device owner). However, the total acquisition (in this case construction) not land purchase costs of each treatment device have been shown in Table 16.

The calculated life cycle costs for each of the stormwater quality treatment devices to be maintained by Council are shown in Table 16.

Table 16 Treatment Train Life Cycle Costs

Cost Component	Bioretention Basin
Total Acquisition (construction) Cost	$85,035
Annual Maintenance Cost	$8,376
Renewal/Adaption Cost	$38,134
Decommissioning Cost	$38,124
Life Cycle Cost ($2011)	**$245,978**
Equivalent Annual Payment Cost of the Asset ($2011/annum)	$4,920
Equivalent Annual Payment /kg Total Suspended Solids/annum	$0.4
Equivalent Annual Payment /kg Total Phosphorus/annum	$261.83
Equivalent Annual Payment /kg Total Nitrogen/annum	$95.77
Equivalent Annual Payment /kg Gross pollutant/annum	$2.77

The relative distribution of all costs elements in the life cycle costs, for each treatment device has been provided in Appendix C.

6 CONCLUSION

It is proposed to include stormwater detention and water quality treatment devices as part of the overall stormwater management system for Stages 2A, 2B, 3A, 3B and 3C. Modelling of the performance of the system has indicated that the design criteria adopted for the Stages will be satisfied.

Figures

LEGEND

- Basin SC2 Catchments
- Development Layout
- XP SWMM Model

75 0 75 150 225 300 375m
1:7500

Scale 1:7,500 (A3)

FIGURE 1

DETENTION BASIN CATCHMENT AREAS

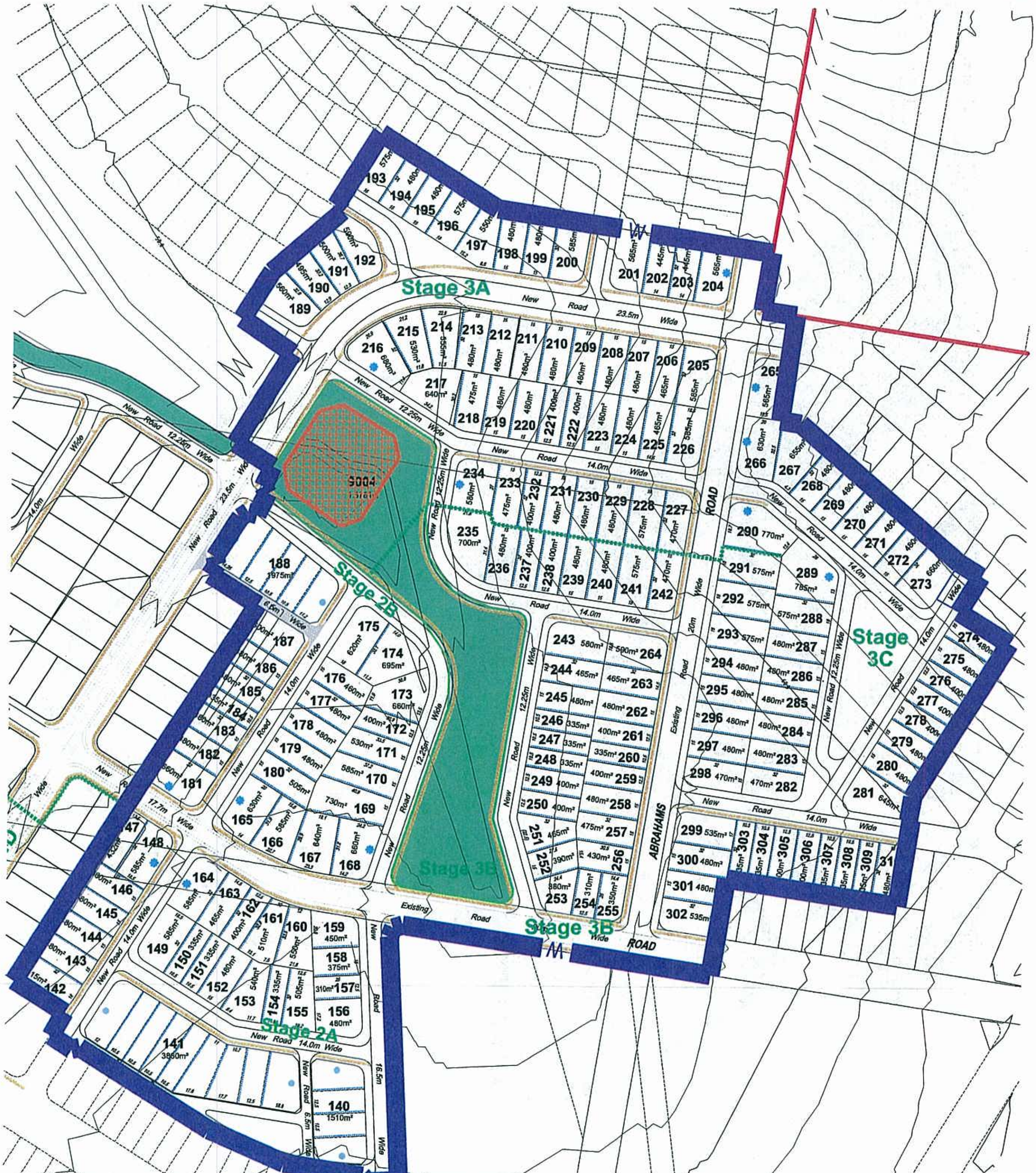
Rev: V3. Date: March 2011

Project No.: 7901-47

Amex Corporation

PRINT DATE: 21 March, 2011 - 11:30am

CAD FILE: J:\7901-47-22\ACAD\Figures - SWMP Precinct C\Figure 1 - Detention Basin Catchment Areas_v3.dwg
XREF's



LEGEND

Catchment Boundary

Sedimentation Basin

30 0 30 60 90 120 150m
1:3000

Scale 1:3,000 (A4)

FIGURE 2
Sedimentation Basin

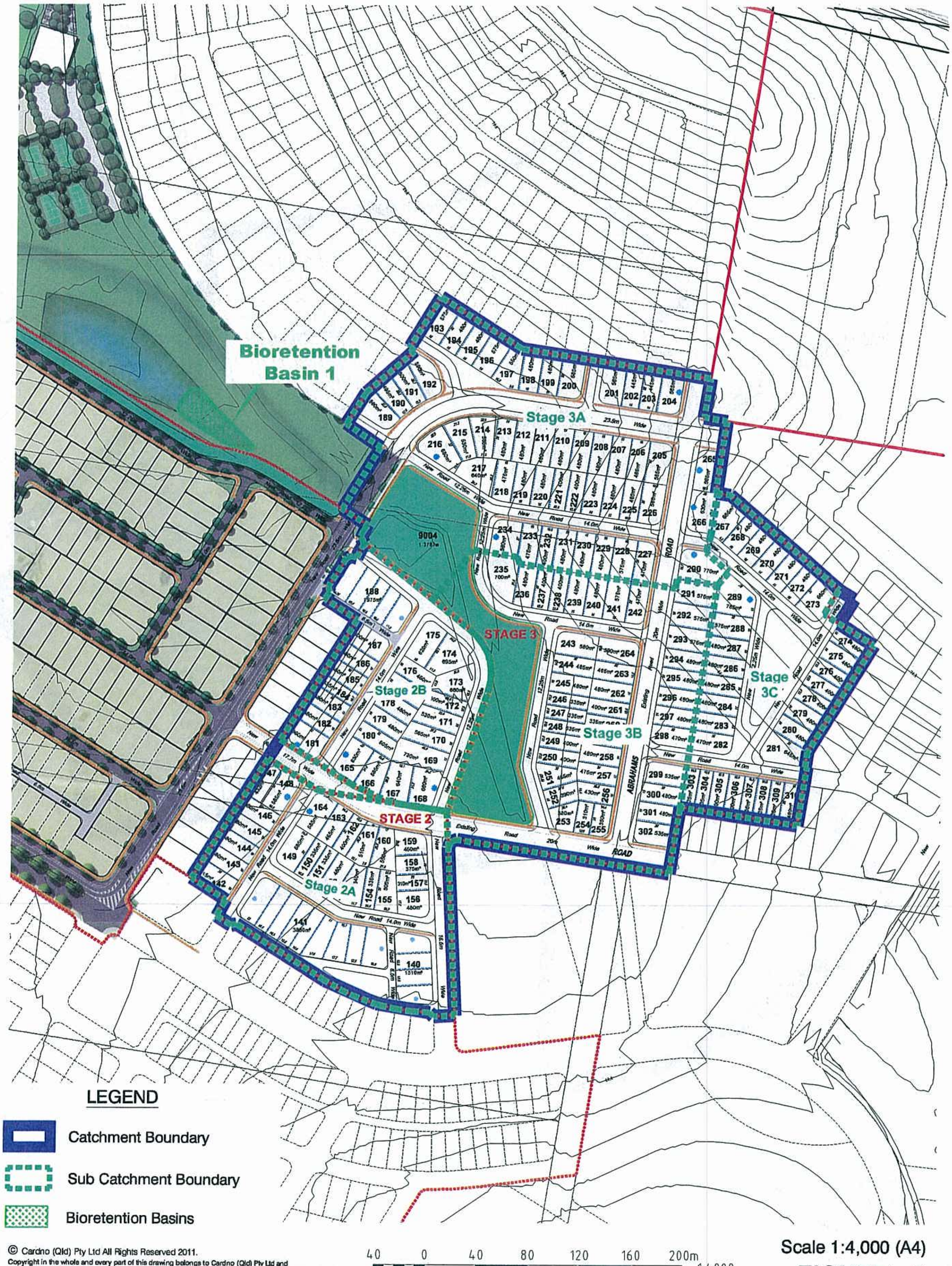
Rev: Orig. Date: March 2011

Project No.: 7901/47

AMEX Coorporation Pty Ltd

PRINT DATE: 21 March, 2011 - 10:50am

CAD FILE: J:\7901-47-22\ACAD\Figures - SWMP Precinct C\Figure 2 - Sedimentation Basins-V3.dwg
XREF's:



Scale 1:4,000 (A4)

FIGURE 3

MUSIC Model Catchment Areas

Rev: V3. Date: March 2011

Project No.: 7901/47

AMEX Coorporation Pty Ltd

PRINT DATE: 21 March, 2011 - 11:11am

CAD FILE: J:\7901-47-22\ACAD\Figures - SWMP Precinct C\Figure 3 - MUSIC Model Catchment Areas-V3.dwg
XREF's: XR-DESIGN CONTOURS

Appendix A

XP-SWMM Model Results

TABLE A1
Existing Case- Peak Water Levels and Depths
1, 2, 5, 10 Year Events

Node Name	Invert Level (mAHD)	1 Year Event Max Level (mAHD)	1 Year Event Depth (m)	1 Year Event Crit Durn	2 Year Event Max Level (mAHD)	2 Year Event Depth (m)	2 Year Event Crit Durn	5 Year Event Max Level (mAHD)	5 Year Event Depth (m)	5 Year Event Crit Durn	10 Year Event Max Level (mAHD)	10 Year Event Depth (m)	10 Year Event Crit Durn
Secondary Urban Centre East- Basin SC2													
N-SC2-01a	51.000	51.300	0.300	1 Hour	51.335	0.335	1 Hour	51.367	0.367	1 Hour	51.384	0.384	1 Hour
N-SC2-01b	44.760	45.162	0.402	1 Hour	45.205	0.445	1 Hour	45.259	0.499	1 Hour	45.282	0.522	1 Hour
N-SC2-02	42.000	42.209	0.209	1 Hour	42.256	0.256	1 Hour	42.319	0.319	1 Hour	42.346	0.346	1 Hour
N-SC2-03	41.000	41.404	0.404	1 Hour	41.486	0.486	1 Hour	41.592	0.592	1 Hour	41.647	0.647	1 Hour
N-SC2-04	41.000	41.397	0.397	1 Hour	41.479	0.479	1 Hour	41.585	0.585	1 Hour	41.641	0.641	1 Hour

TABLE A2
Existing Case- Peak Water Levels and Depths
20, 50, 100 Year Events

Node Name	Invert Level (mAHD)	20 Year Event			50 Year Event			100 Year Event		
		Max Level (mAHD)	Depth (m)	Crit Durn	Max Level (mAHD)	Depth (m)	Crit Durn	Max Level (mAHD)	Depth (m)	Crit Durn
Secondary Urban Centre East- Basin SC2										
N-SC2-01a	51.000	51.406	0.406	1 Hour	51.433	0.433	1 Hour	51.455	0.455	1 Hour
N-SC2-01b	44.760	45.309	0.549	1 Hour	45.341	0.581	1 Hour	45.367	0.607	1 Hour
N-SC2-02	42.000	42.378	0.378	1 Hour	42.418	0.418	1 Hour	42.449	0.449	1 Hour
N-SC2-03	41.000	41.652	0.652	1 Hour	41.656	0.656	1 Hour	41.660	0.660	1 Hour
N-SC2-04	41.000	41.645	0.645	1 Hour	41.649	0.649	1 Hour	41.652	0.652	1 Hour

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TABLE A3
Developed Case- Peak Water Levels and Depths
1, 2, 5, 10 Year Events

Node Name	Invert Level (mAHD)	1 Year Event			2 Year Event			5 Year Event			10 Year Event		
		Max Level (mAHD)	Depth (m)	Crit Durn	Max Level (mAHD)	Depth (m)	Crit Durn	Max Level (mAHD)	Depth (m)	Crit Durn	Max Level (mAHD)	Depth (m)	Crit Durn
Secondary Urban Centre East- Basin SC2													
N-SC2-01	46.000	46.427	0.426	25 Min	46.509	0.509	25 Min	46.596	0.596	25 Min	46.644	0.644	25 Min
N-SC2-02	42.500	43.451	0.951	25 Min	43.623	1.123	25 Min	43.802	1.302	25 Min	43.898	1.398	25 Min
N-SC2-03	42.000	42.457	0.457	1 Hour	42.632	0.632	1 Hour	42.861	0.861	1 Hour	42.953	0.953	1 Hour
N-SC2-04	40.500	40.812	0.312	1 Hour	40.832	0.332	1 Hour	40.885	0.385	1 Hour	40.952	0.452	1 Hour
N-SC2-01a	53.500	53.990	0.490	25 Min	54.058	0.558	25 Min	54.127	0.627	25 Min	54.164	0.664	25 Min
N-SC2-02a	51.000	51.511	0.511	25 Min	51.589	0.589	25 Min	51.675	0.675	25 Min	51.725	0.725	25 Min
N-SC2-03a	49.000	49.540	0.540	25 Min	49.604	0.604	25 Min	49.661	0.661	25 Min	49.685	0.685	25 Min
N-SC2-01b	47.493	47.794	0.301	25 Min	47.901	0.408	25 Min	48.032	0.539	25 Min	48.110	0.617	25 Min
Outfall	40.400	40.709	0.309	1 Hour	40.730	0.330	1 Hour	40.784	0.384	1 Hour	40.851	0.451	1 Hour
N_BasinEnd	41.700	42.434	0.733	1 Hour	42.624	0.924	1 Hour	42.857	1.157	1 Hour	42.947	1.247	1 Hour
N_Sports	43.900	43.900	0.000	15 Min	43.900	0.000	15 Min	43.900	0.000	15 Min	43.900	0.000	15 Min
N_BasinMid	41.840	42.439	0.599	1 Hour	42.626	0.786	1 Hour	42.858	1.018	1 Hour	42.949	1.109	1 Hour

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TABLE A4
Developed Case- Peak Water Levels and Depths
20, 50, 100 Year Events

Node Name	Invert Level	20 Year Event			50 Year Event			100 Year Event		
		Max Level	Depth	Crit Durn	Max Level	Depth	Crit Durn	Max Level	Depth	Crit Durn
	(mAHD)	(mAHD)	(m)		(mAHD)	(m)		(mAHD)	(m)	
Secondary Urban Centre East- Basin SC2										
N-SC2-01	46.000	46.703	0.703	25 Min	46.739	0.739	25 Min	46.774	0.774	20 Min
N-SC2-02	42.500	44.015	1.515	25 Min	44.100	1.600	25 Min	44.195	1.695	25 Min
N-SC2-03	42.000	43.046	1.046	1 Hour	43.153	1.153	1 Hour	43.229	1.229	1 Hour
N-SC2-04	40.500	41.036	0.536	1 Hour	41.142	0.642	1 Hour	41.221	0.721	1 Hour
N-SC2-01a	53.500	54.211	0.711	25 Min	54.240	0.740	25 Min	54.280	0.780	25 Min
N-SC2-02a	51.000	51.790	0.790	25 Min	51.828	0.828	25 Min	51.878	0.878	25 Min
N-SC2-03a	49.000	49.714	0.714	25 Min	49.746	0.746	25 Min	49.788	0.788	25 Min
N-SC2-01b	47.493	48.215	0.722	25 Min	48.289	0.795	25 Min	48.434	0.941	25 Min
Outfall	40.400	40.935	0.535	1 Hour	41.041	0.641	1 Hour	41.121	0.721	1 Hour
N_BasinEnd	41.700	43.039	1.339	1 Hour	43.143	1.443	1 Hour	43.218	1.518	1 Hour
N_Sports	43.900	43.900	0.000	15 Min	43.900	0.000	15 Min	43.900	0.000	15 Min
N_BasinMid	41.840	43.042	1.202	1 Hour	43.147	1.307	1 Hour	43.223	1.383	1 Hour

TABLE A5
Peak Flows and Velocities, 1, 2 and 5 Year Events - Existing Case

Link Name	1 Year Event			2 Year Event			5 Year Event		
	Existing			Existing			Existing		
	Max Flow	Crit Durn	Max Vel	Max Flow	Crit Durn	Max Vel	Max Flow	Crit Durn	Max Vel
Secondary Urban Centre East- Basin SC2									
I-SC2-01a	2.71	1 Hour	0.57	3.87	1 Hour	0.62	5.60	1 Hour	0.68
I-sc2-01	3.97	1 Hour	0.45	5.55	1 Hour	0.49	8.10	1 Hour	0.55
I-sc2-02	5.46	1 Hour	0.43	7.69	1 Hour	0.46	11.02	1 Hour	0.50
I-sc2-03	6.00	1 Hour	0.09	8.35	1 Hour	0.10	11.63	1 Hour	0.10

TABLE A6
Peak Flows and Velocities, 10, 20 and 50 Year Events -Existing Case

Link Name	10 Year Event			20 Year Event			50 Year Event		
	Existing			Existing			Existing		
	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)
l-SC2-01a	6.62	1 Hour	0.70	7.99	1 Hour	0.74	9.81	1 Hour	0.78
l-sc2-01	9.66	1 Hour	0.58	11.64	1 Hour	0.61	14.25	1 Hour	0.65
l-sc2-02	13.22	1 Hour	0.52	15.94	1 Hour	0.54	19.69	1 Hour	0.58
l-sc2-03	14.56	1 Hour	0.11	18.66	1 Hour	0.11	22.95	1 Hour	0.12

TABLE A7
Peak Flows and Velocities, 100 Year Event

Link Name	100 Year Event		
	Existing		
	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)
Secondary Urban Centre East- Basin SC2			
I-SC2-01a	11.32	1 Hour	0.81
I-sc2-01	16.46	1 Hour	0.69
I-sc2-02	22.73	1 Hour	0.62
I-sc2-03	26.47	1 Hour	0.13

TABLE A8
Peak Flows and Velocities, 1, 2 and 5 Year Events

Link Name	1 Year Event			2 Year Event			5 Year Event		
	Developed			Developed			Developed		
	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)
Secondary Urban Centre East- Basin SC2									
Sect A	7.22	25 Min	1.281	9.82	25 Min	1.400	12.89	25 Min	1.510
Sect B	7.02	25 Min	1.528	9.61	25 Min	1.704	12.70	25 Min	1.862
Sect C	6.89	25 Min	1.029	9.46	25 Min	1.157	12.52	25 Min	1.304
Chan_1	9.83	25 Min	1.072	13.46	25 Min	1.209	17.83	25 Min	1.345
Chan_2	11.92	25 Min	0.548	16.66	25 Min	0.631	22.54	25 Min	0.717
DNST Link	5.48	1 Hour	0.563	6.09	1 Hour	0.586	7.80	1 Hour	0.642
Det Basin	12.62	1 Hour	0.273	17.43	25 Min	0.317	23.70	25 Min	0.361
SportField	0.00	15 Min	0.000	0.00	15 Min	0.000	0.00	15 Min	0.000
DetBasin_2	8.80	25 Min	0.219	11.85	25 Min	0.249	15.41	25 Min	0.278
lnk234pipe	1.42	25 Min	3.787	1.94	25 Min	3.880	2.57	25 Min	3.944
Rd234wr	0.00	15 Min	0.000	0.00	15 Min	0.000	0.00	15 Min	0.000
Weir	0.00	15 Min	0.000	0.00	15 Min	0.000	1.19	1 Hour	0.265
P_Outlet_2	1.10	1 Hour	4.319	1.22	1 Hour	2.337	1.32	1 Hour	2.611
FREE # 1	5.48	1 Hour	0.000	6.09	1 Hour	0.000	7.80	1 Hour	0.000

Link Name	10 Year Event			20 Year Event			50 Year Event		
	Developed			Developed			Developed		
	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)
Sect A	14.74	25 Min	1.564	17.26	25 Min	1.632	18.95	25 Min	1.671
Sect B	14.50	25 Min	1.923	16.97	25 Min	1.998	18.74	25 Min	2.061
Sect C	14.34	25 Min	1.388	16.82	25 Min	1.496	18.58	25 Min	1.524
Chan_1	20.45	25 Min	1.417	23.97	25 Min	1.506	26.78	25 Min	1.593
Chan_2	26.13	25 Min	0.763	30.86	25 Min	0.817	34.57	25 Min	0.855
DNST Link	10.22	1 Hour	0.711	13.64	1 Hour	0.792	18.47	1 Hour	0.884
Det Basin	27.45	25 Min	0.384	32.26	25 Min	0.412	35.85	20 Min	0.432
SportField	0.00	15 Min	0.000	0.00	15 Min	0.000	0.00	15 Min	0.000
DetBasin_2	17.28	25 Min	0.293	19.71	25 Min	0.310	22.00	20 Min	0.326
Ink234pipe	2.95	25 Min	3.971	3.45	25 Min	4.004	3.80	25 Min	4.010
Rd234wr	0.00	15 Min	0.000	0.00	15 Min	0.000	0.00	15 Min	0.000
Weir	3.06	1 Hour	0.425	5.88	1 Hour	0.585	10.17	1 Hour	0.761
P_Outlet_2	1.43	1 Hour	2.794	1.55	1 Hour	9.755	1.66	1 Hour	3.142
FREE # 1	10.22	1 Hour	0.000	13.64	1 Hour	0.000	18.47	1 Hour	0.000

TABLE A10
Peak Flows and Velocities, 100 Year Event

Link Name	100 Year Event		
	Developed		
	Max Flow (m^3/s)	Crit Durn (-)	Max Vel (m/s)
Secondary Urban Centre East- Basin SC2			
Sect A	21.38	25 Min	1.725
Sect B	21.18	25 Min	2.140
Sect C	21.02	25 Min	1.546
Chan_1	29.72	25 Min	1.693
Chan_2	38.99	25 Min	0.897
DNST Link	22.46	1 Hour	0.947
Det Basin	40.35	20 Min	0.454
SportField	0.00	15 Min	0.000
DetBasin_2	24.20	20 Min	0.340
Ink234pipe	4.21	25 Min	4.021
Rd234wr	0.00	15 Min	0.000
Weir	13.98	1 Hour	0.883
P_Outlet_2	1.70	1 Hour	3.179
FREE # 1	22.46	1 Hour	0.000